Zeitschrift: IABSE congress report = Rapport du congrès AIPC = IVBH Kongressbericht

Band: 2 (1936)

Artikel: New dry-docks in the harbours of Genoa and Naples

Autor: Krall, G. / Straub, H.

DOI: https://doi.org/10.5169/seals-3217

VI 4

New Dry-Docks in the Harbours of Genoa and Naples.

Neue Trockendocks in den Häfen von Genua und Neapel.

Nouvelles cales sèches dans les ports de Gênes et Naples.

Professor Ing. G. Krall und Dipl.-Ing. H. Straub,
Rom.

The design of the two large new dry docks in the harbours of Genoa and Naples — the construction of which has since been put in hand, and which will be among the largest and boldest structures of their kind — gave rise to various statical problems which are of general interest.

The transverse dimensions of both works are the same: clear width between side walls 40.0 m depth of dock floor below mean water level 14 m: thickness of side walls 9.0 m — see Figs. 1—3.

The statical behaviour of the two works, however, are fundamentally different, the dock at Genoa being founded on rock while that at Naples rests on a sandy bottom.

A. *Dry dock at Genoa.*

In order to limit the amount of work having to be performed with the aid of compressed air the following working procedure was adopted. The side walls and end wall are built first, together with the connecting groove for the end sill. After the floating caisson gate has been placed in position the dock space is closed on all sides and can be pumped out, allowing the floor to be concreted and the masonry lining to be constructed in the dry.

This method of construction, however, has the disadvantage that pending the construction of the floor, the side walls have to carry the whole of the external water pressure directly on to the rock bottom, since during this period the dock floor is not available to buttress the two side walls against one another as in the finished structure. Since, in the case of the new dry dock at Genoa, the sound rock lies some 5—7 m below the level of the future floor, the loading of the side walls during this place of construction is by far less favourable than in the finished work, a circumstance which gave occasion for the method of calculation described below.

The end wall and side walls are made up of reinforced concrete caissons sunk to impermeable rock of adequate bearing power with the aid of compressed air. For the reasons mentioned above the arrangement of caissons shown in Figs. 4 and 5 has been adopted, alternate caissons being placed lengthwise and

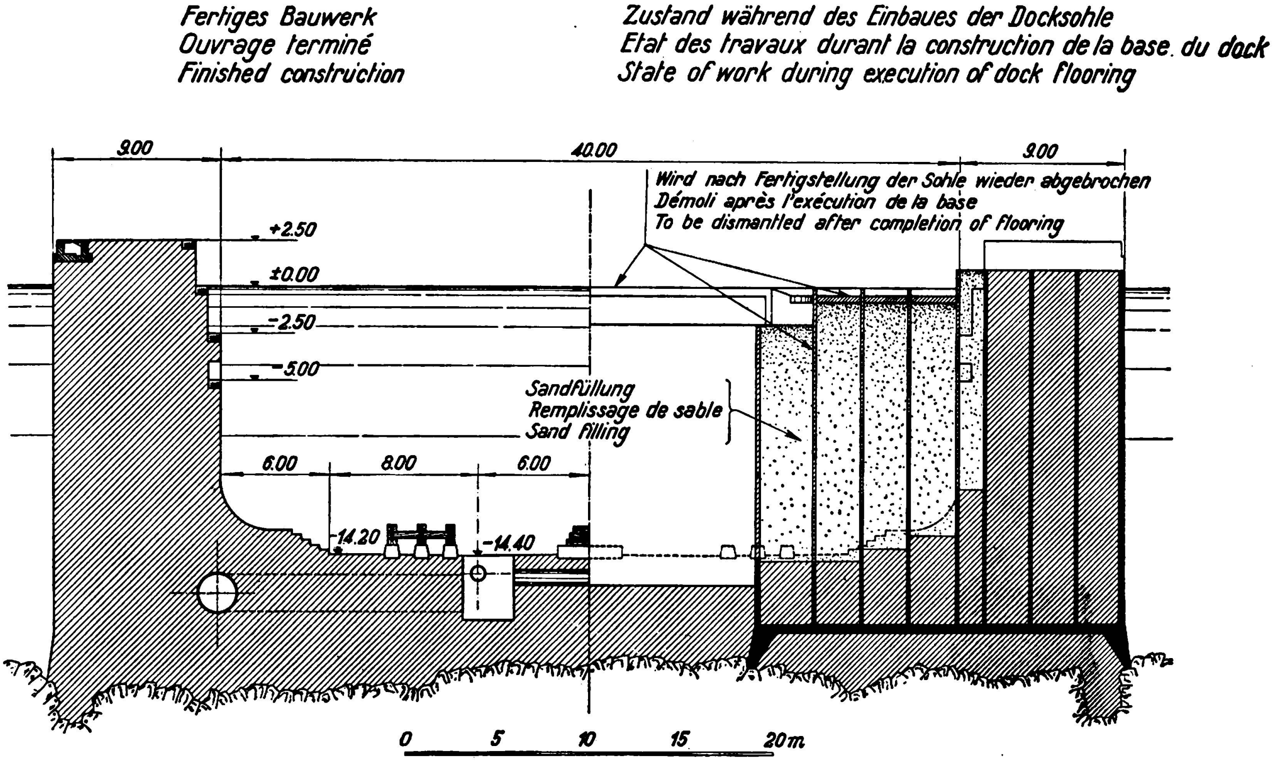

Fig. 1.
Cross section of dry dock in Genoa.

crosswise so that together they form a counterforted dam. In the caissons placed lengthwise, the intermediate walls are so planned that when certain of the cells are filled with stronger concrete they form a horizontal arch. The transverse caissons act as counterforts. In addition each pair of opposite piers is connected above by a strut. After the floor of the dock has been completed these struts and those portions of the counterforts which project inside are cut away.

It ist evident that during the phase of construction now considered, part of the water pressure is borne by the side wall fixed into the rock, while another

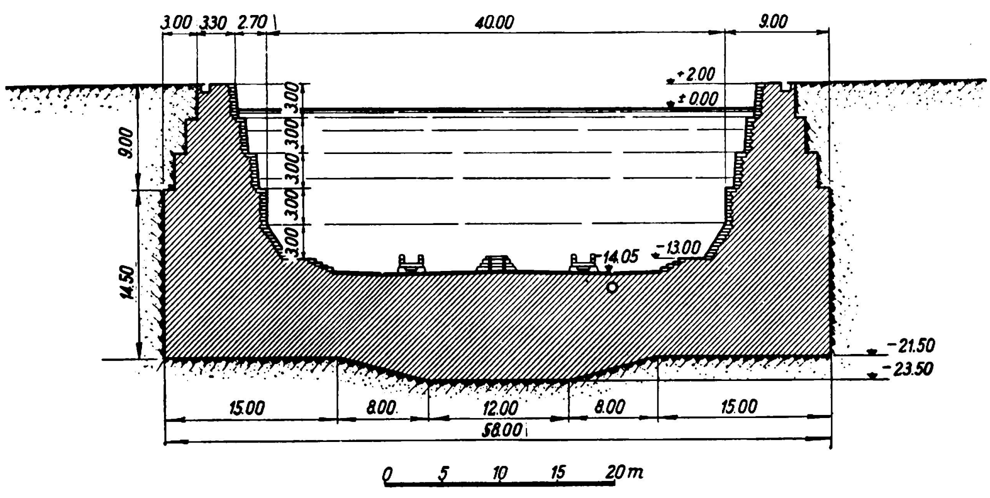

Fig. 2.
Cross section of dry dock in Naples.

part, however, is carried to the counterforts by arch action. In order to determine the stability it is necessary to the probable distribution of the pressure between these two systems[1].

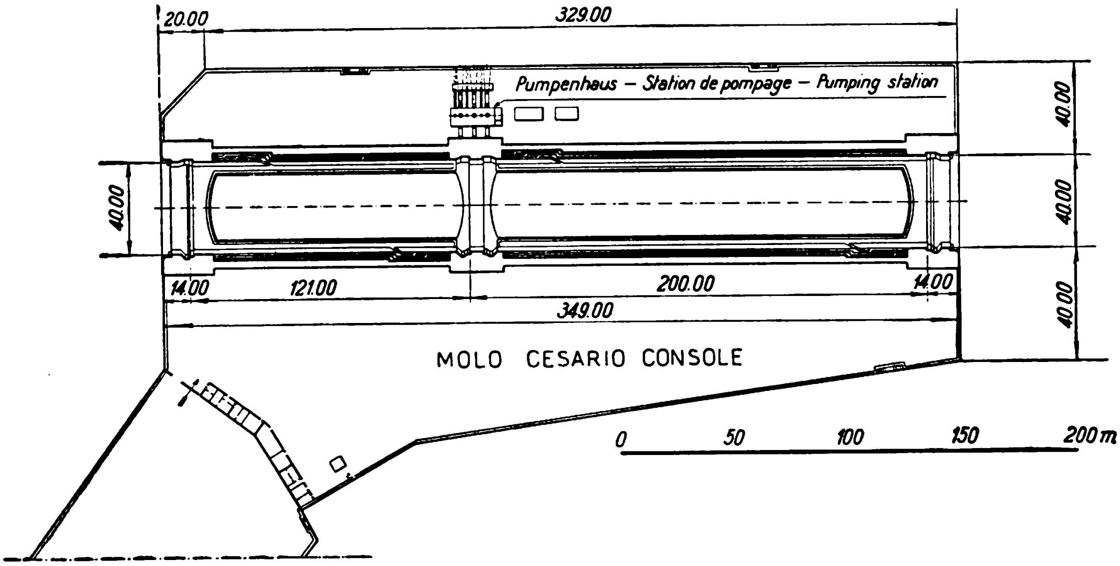

Fig. 3.

Plan of dry dock in Naples.

If p_o denote the maximum water pressure arising at the point of fixation, the following expression may be given for the share transmitted by the side wall direct on to the rock (see Fig. 4):

$$p'(x, y) = p_o \left(\frac{x}{h}\right)^m \cdot \frac{1 - \cos\frac{2\pi}{l} \cdot y}{2} \qquad \text{(See Fig. 6)} \qquad (1)$$

The approximately sinusoidal form of the surface p in the horizontal direction can be inferred straight away from the mode of action of the dam as here explained, and any such small deviation from this shape as may occur has scarcely any effect on the final result.

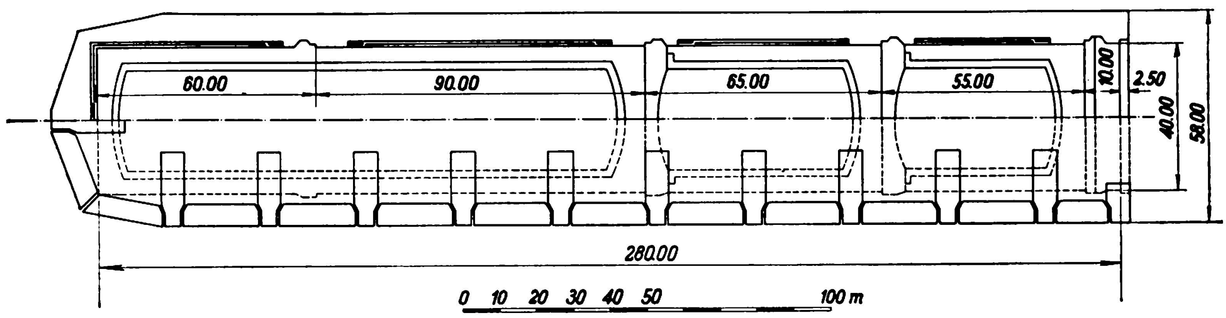

Fig. 4.

Plan of dry dock in Genoa.

[1]) Cf. *K. Krall:* Problemi inerenti alla costruzione del nuovo Bacino di Carenaggio a Genova: Annali dei Lavori Publici 1935, Fasc. II Roma.

The assumption of a parabolic form in the vertical plane enables the problem to be handled mathematically. The nature of the division of pressure between the wall and the counterforts, which governs the conditions of stability, is completely expressed by the magnitude of the exponent m.

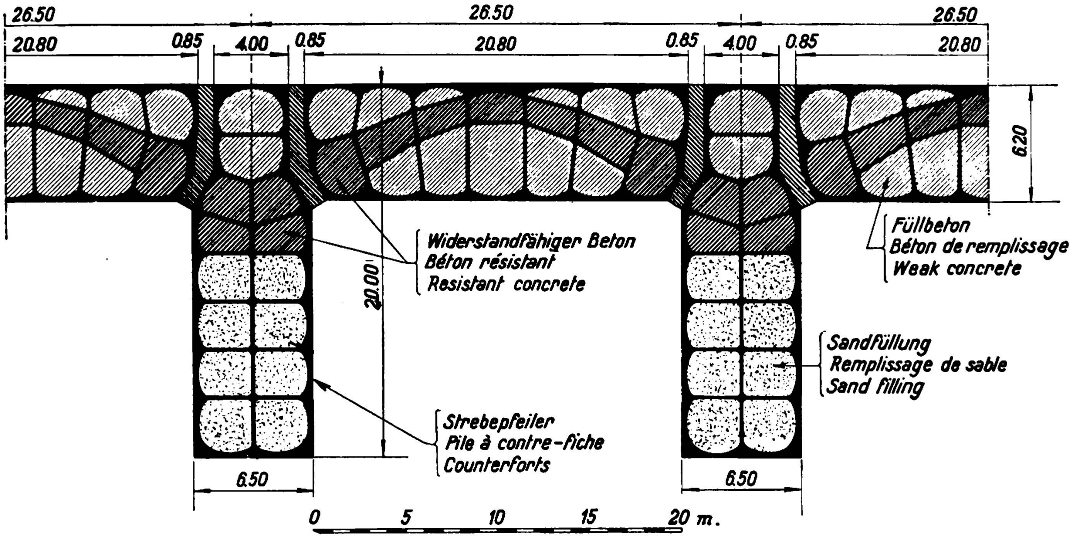

Fig. 5.

Dry dock Genoa: Layout of caissons.

The solution is obtained in the normal way by equating the displacements of the two elastic systems, in the form:

$$\varphi (m) = \psi [X (m); m] \tag{2}$$

Where X denotes the force in the upper strut and where the condition is to be fulfilled that equality exists —

I. between half the shortening of the strut, and the displacement of the upper edge of the counterfort,

and II. between the flexure of the central strip of wall which is built in at the foot, and the displacement of the crown of the arch bearing on to the counterforts.

The latter equation must be accurately true for a point at middle height; it may then be shown as a corollary that approximate equality will obtain over the whole height.

To solve the problem mathematically, formulae were next developed for the deflections and elastic lines, the following separate determinations being made:

1) The elastic line for the strip of wall elastically fixed at the foot, under the loading $p_0 \left(\frac{x}{h}\right)^m$.

2) The crown displacement of the arch under the variable load $p_0 \cdot \frac{x}{h} - p'(x, y)$ the supports being assumed invariable.

3) The elastic line of the counterfort built in at its foot, the load thereon being compounded of the bearing reactions of the adjacent arches, the full water pressure over the width of the counterfort itself, and the reaction X in the strut. (Here the counterfort was first assumed to be rigidly fixed at its foot).

4) The additional rotation of the counterfort due to the elastic yield occurring in the cross section built in.

5) The shortening of the strut under the load X.

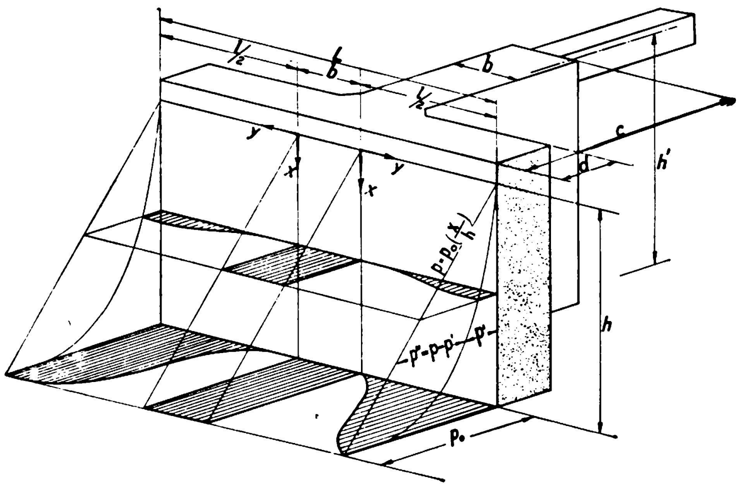

Fig. 6.

Dry dock Genoa: Distribution of water pressure on longitudinal walls and buttresses.

In these expressions the flexures appear as functions of the amount m, the load in the strut X, and, in reference to elastic lines, the ordinate x. The dimensions of the individual constructional members, the moduli of elasticity of the materials, and the ground constant or bedding number C, enter as constants.

If the several displacements are denoted respectively by $w_1, \ldots\ldots, w_5$ in the order indicated above, the critical equations assume the following forms:

$$\frac{1}{2} w_5 = w_3 (h') + w_4 (h') \qquad (3)$$

and

$$w_1 (x) = w_2 (x) + w_3 (x) + w_4 (x) \qquad (4)$$

When $x = \frac{1}{2} h$ is inserted in (4) as stated above, the only unknowns remaining in these two equations are the quantities m and X, and the solution corresponds with the statement (2) as already given:

$$\varphi (m) = \psi [X (m); m]$$

where

$$\varphi\ (m) = \frac{h^4}{E_m\, i}\left\{\frac{\left[\frac{m}{2}+1+\left(\frac{1}{2}\right)^{m+4}\right]}{(m+1)\ (m+2)\ (m+3)\ (m+4)}+\frac{E_m}{2\,C\,h}\,\frac{1}{(m+1)\ (m+2)}\right\}$$

$$\Psi\,[X\,(m);\,m] = \frac{l^4\left[1+\left(\frac{f}{a}\right)^2\right]}{64\,f^2\,E_a\cdot s}\left[1-0{,}6\left(\frac{1}{2}\right)^{m-1}\right]$$

$$+\frac{h^4\,L}{120\,E_m\,I}\left\{1{,}50+\left(\frac{1}{2}\right)^4-\frac{1}{2\,L}\cdot\frac{120\left[\frac{m}{2}+1+\left(\frac{1}{2}\right)^{m+4}\right]}{(m+1)\,(m+2)\,(m+3)\,(m+4)}\right.$$

$$\left.-\frac{X}{L\,h}\left[60\,\frac{E_m}{C\,h}\cdot\frac{h'}{h}+40\left(1-\frac{3}{2}\left(\frac{x'}{h'}\right)+\frac{1}{2}\left(\frac{x'}{h'}\right)^3\right)\right]\right\}+\frac{10\,E^m}{C\,h}\left(1-\frac{1}{2\,L}\cdot\frac{6}{(m+1)\,(m+2)}\right)$$

$$X\,(m) = \frac{1}{30}\left(\frac{h}{h'}\right)^2\frac{C\,h}{E_m}\,(L\cdot h)\left\{\frac{1-\frac{1}{2\,L}\cdot\frac{30\left[(m+3)-(m+4)\,\frac{h-h'}{h}\right]}{(m+1)\,(m+2)\,(m+3)\,(m+4)}}{1+\frac{I}{F_p\,h'^2}-\frac{D\,C}{2\,E_p}+\frac{C\,h'}{3\,E_m}}\right.$$

$$\left.+\frac{\frac{5\,E_m\,\frac{h'}{h}}{C\,h}\left(\frac{1}{2\,L}\cdot\frac{6}{(m+1)\,(m+2)}\right)}{1+\frac{I}{F_p\,h_1^{\,2}}-\frac{D\,C}{2\,E_p}+\frac{C\,h'}{3\,E_m}}\right\}$$

Here E_m = modulus of elasticity of caisson plus concrete filling (mean value)
E_a = modulus of elasticity of horizontal arch
E_p = „ „ „ „ strut
I = moment of inertia of effective cross section of counterfort
i = „ „ „ „ strip of wall of unit length = $\frac{d^3}{12}$
F_p = cross section of strut
s = thickness of arch at crown
f = "rise" of horizontal arch
a = $\frac{l}{2}$
D = length of strut
h' = heigth of axis of strut above built-in cross section of counterfort
x' = $x + (h' - h)$

The numerical calculation is made by taking a number of values of m (m = 2, 3, 4....) and finding first the reaction X (m = 2, 3, 4....) then the functions φ (m = 2, 3, 4....) and ψ (m = 2, 3, 4....). If now, the curves φ (m) and ψ (m) are plotted on a co-ordinate system, the required value m will be determined by their intersection.

In the actual case of the dry dock at Genoa it was found that m = ∼ 6; hence, in accordance with the conditions of execution and the available qualities of concrete, adopted in the different parts of the structure the following constants were adopted for the calculations:

$$\text{Moduli of elasticity} \begin{cases} \text{for the wall built in at foot } E_m = 1.2 \times 10^6 \text{ tons per sq. cm.} \\ \text{for the arch, } E_a = 1.8 \times 10^6 \text{ tons per sq. cm.} \\ \text{for the strut, } E_p = 2.2 \times 10^6 \text{ tons per sq. cm.} \end{cases}$$

For the ground constant C, in order to be on the safe side a considerably lower value was chosen than might be expected to hold good in reality, namely

$$C = 15\,\text{kg/cm}^3 = 1.5 \cdot 10^4\,\text{tm}^3$$

No experimental values for C are available in respect of bearing areas as large as those here considered (approximately 6.6 m $\times$ 20.0 m. If, conversely, C be assumed as proportional to the width of the loaded surface — as is no doubt approximantely correct in the case of a rock foundation[2] — then the adopted value C = 15 corresponds to a value of about 300 kg/cm² on a surface of the dimensions usual in experiments (about 1.00 m $\times$ 0.30 m.

Construction.

The period allowed for construction is the relatively short one of $3^1/_2$ years. In fixing this it was, of course, a matter of the greatest importance that it should be possible to prepare the 47 necessary caissons, having the notable dimensions indicated in Fig. 7, in the shortest possible time.

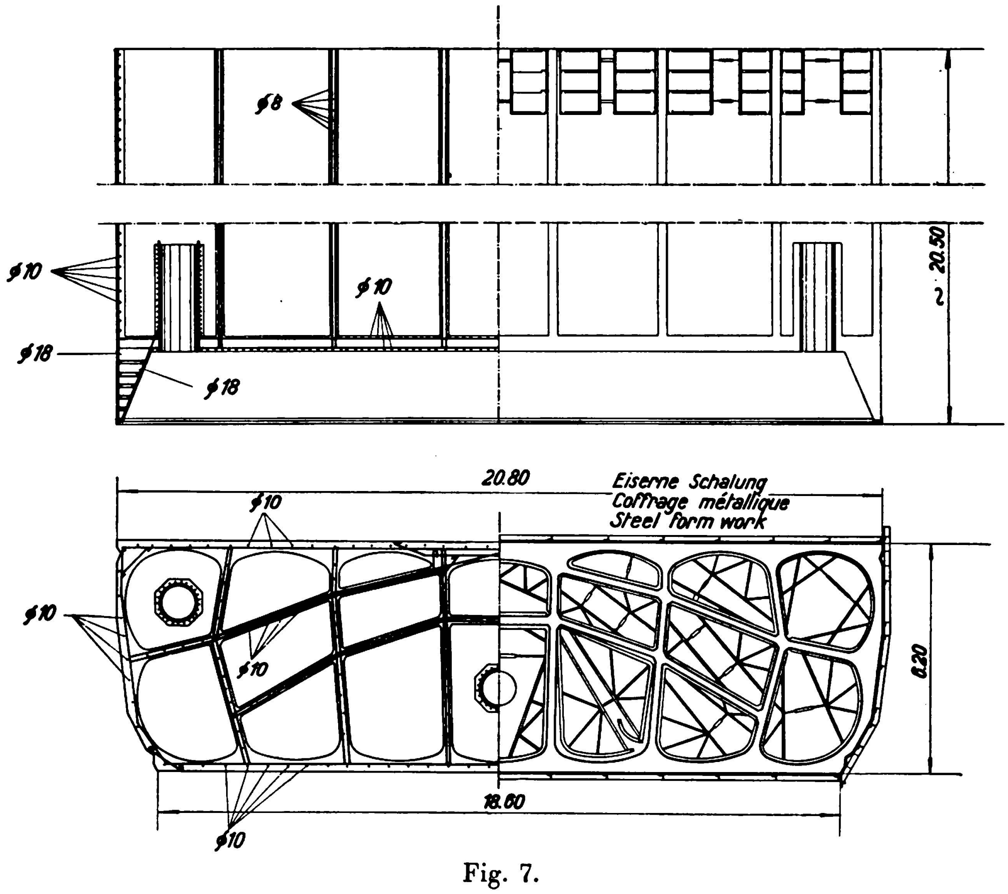

Fig. 7.

Dry dock Genoa: Details of a caisson.

[2]) cf. *Schleicher:* Bauingenieur 1926, p. 931,

For this purpose the joint contractors (SILM-Società Italiana per Lavori Marittimi and SIFC-Società Italiana Finanziaria per Costruzioni) is making use of two small docks specially built for the construction of reinforced concrete floating bodies (Fig. 8). The working chamber and the cells above it to a height of 11.0 m are formed in these, then the floating caissons are towed to their future sites, where the cell walls are continued upward to the necessary height of 18—20 m.

In order to accelerate the work to the utmost and ensure that the concrete surface shall be as smooth and impermeable as possible steel forms are used

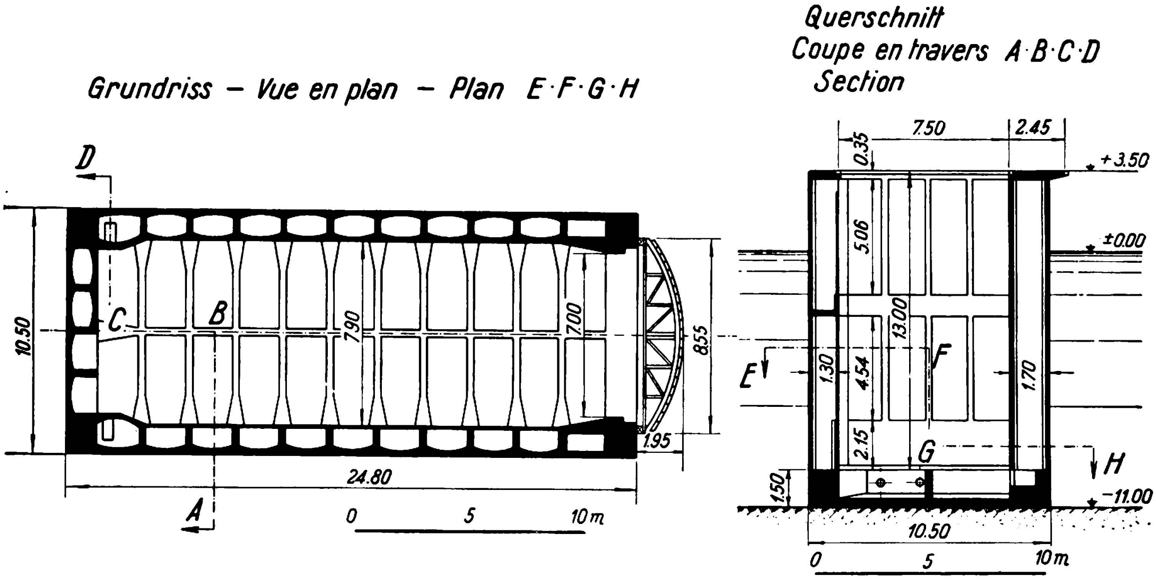

Fig. 8.

Dry dock Genoa: Dock chamber for manufacturing caissons.

(see Fig. 7). Special appliances are adopted whereby the forms may be stripped, raised and refitted within the short period of about six hours, and in this way it is possible to complete one caïsson in each of the two docks in 15—20 days (Fig. 9).

For building the caissons, use is made of a plastic concrete containing 300 kg of cement per m^3 of finished concrete. With a view to greater resistance of the concrete to deleterious effects of the sea water, what is known as pozzolanic cement was chosen instead of ordinary Portland cement. Recently in Italy this material has been much used, with success, for maritime and harbour works; it is a cement poor in lime, and in the course of its manufacture a certain percentage of pozzolanic earth is added to the clinker before grinding, having, like trass, the property of combining with any lime that may be liberated.

The cement adopted has a standard strength of 450 kg/cm^2. Since the conclusion of the experiments made during the first few months to determine the best aggregates and most favourable proportions, it has been possible to obtain with this a briquette strength of 250 kg/cm^2 in concrete 28 days old (Fig. 10).

At the place of sinking, the sea bottom is first dredged down to the rock and is then levelled with the aid of a diving bell suspended from a float; the

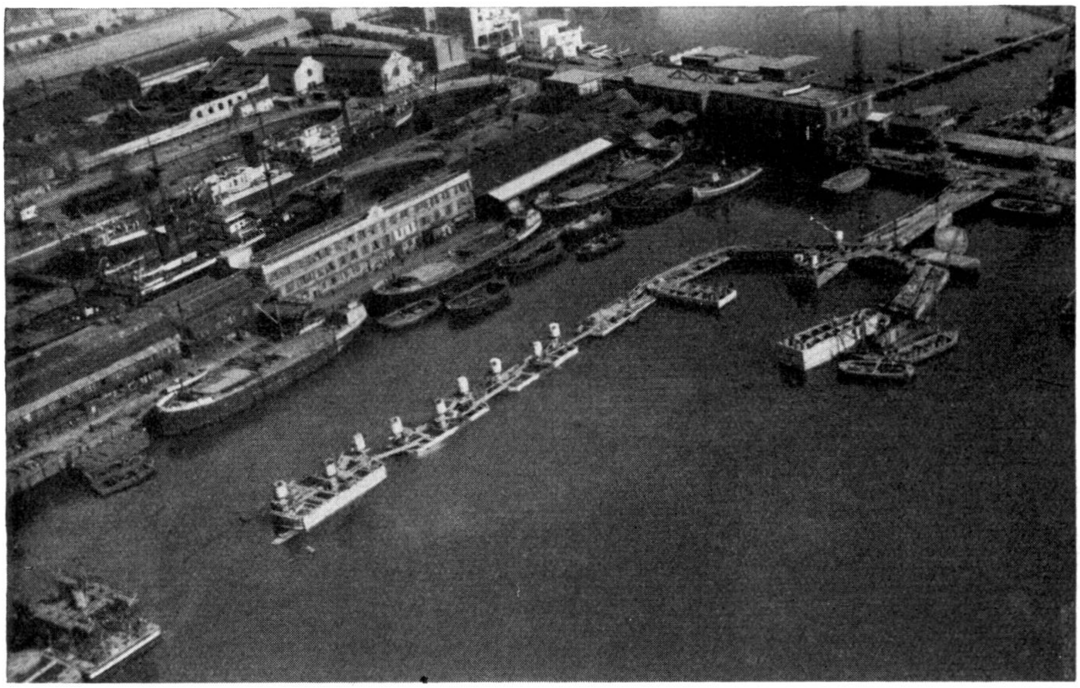

Fig. 9.
Dry dock Genoa: State of work March 1936.

caisson is bedded in its correct position by adding ballast, and is sunk, by the use of compressed air, on to sound rock free absolutely from fissures. The greatest depth reached by any of the caissons sunk up to the present time is — 23.65 m measured from the cutting edge to mean water level. In order to form the waterproofing apron along the foot of the outside of the wall it was necessary to carry the rock excavation a few metres deeper than the cutting edge.

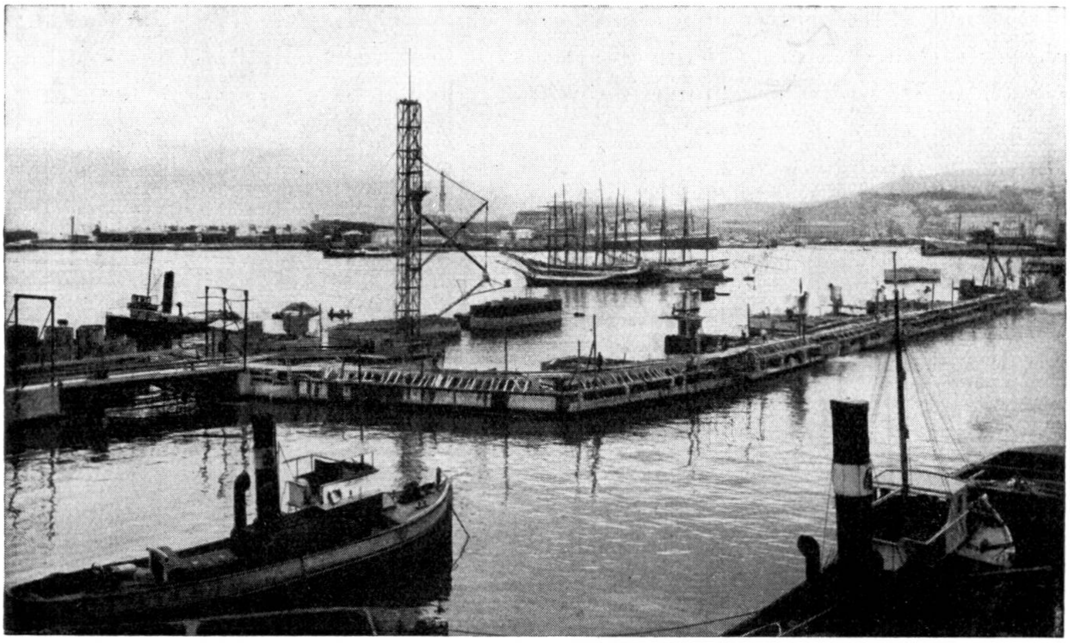

Fig. 10.
Dry dock Genoa: State of work May 1936.

In the case of a few of the pairs of counterforts, the depths actually reached exceed those previously assumed in the calculations by a not inconsiderable amount. In these instances, to restore the conditions underlying the calculations, the opposite counterforts have been buttressed against one another by ground sills in addition to the struts above. These sills are of the same width as the counterforts themselves and their thickness varies according to the depth of the sound bedrock. These again were constructed with the aid of the diving bell mentioned above.

B) Dry dock at Naples.

In contrast to the conditions existing at Genoa, the site for the new dry dock at Naples consists of sand partly interpenetrated by clay, mud and pumice. In broad outline, the method of construction proposed by the contractors (SILM-Società Italiana per Lavori Marittimi) is as follows:

After the bottom has been dredged to the necessary depth two service of reinforced concrete bridges are built outside of and parallel to the side walls of the dock, and two moving steel gantries, each of 68 m span are erected between them to carry the compressed air caissons wherewith the whole of the concrete work under water is done. The latter is formed with a gap separating the floor of the dock from the two side walls so that the settlement of each of these three parts may take place independently. The gap is closed, once more by the use of diving bells, only when no further settlements can be detected.

The cross sections — especially as regards the thickness of the floor — are so dimensioned that tensile stresses cannot arise under any condition of loading; hence steel reinforcements can be dispensed with. The maximum compressive stresses are so low (about 8 kg/cm^2 that instead of cement concrete a mixture of crushed rock lime and pozzolanic earth can be used. Apart from the cheapness of this material — for the most productive sources of pozzolana in Italy are in the immediate neighbourhood of Naples — it offers the advantage of great resistance to the chemical effects of sea water. In the Gulf of Naples there still exist ruins of submarine works constructed with it at the time of the Roman Empire.

In the circumstances described, it was doubly necessary to obtain a correct understanding of the statical behaviour of the structure taking account of the elastic yield both of the material and the foundation. In addition, therefore, to the usual calculation made by the line of pressure method, a further check was carried out in accordance with the procedure briefly described below[3].

For the purpose of analysis the structure is considered as being divided by two planes of section into three parts, as represented diagrammatically in Fig. 11. In this way the portions corresponding to the side walls may be conceived as rigid blocks elastically supported on two sides, while the middle portion may be conceived as an elastic beam. It is required to determine the reactions at the sections (longitudinal and transverse forces and bending moment) complying with the condition that the displacements on each side of the dividing surfaces must balance one another.

---

[3]) cf. *G. Krall:* Problemi statici delle Costruzioni Marittime Reale Accademia d'Italia, Memorie della classe di sienze fisiche, matematiche e naturali, volume V, 1933.

It becomes evident that the problem can most simply be solved by having recourse to the theory of the ellipse of stress. For this purpose such ellipses are drawn in respect of the two dividing surfaces, one for the side block, one for the central beam. The former being rigid on elastic supports the elements of the ellipse (i. e. its centre, diameter and elastic weight) are determined by the established method (see W. Ritter: Anwendungen der graphischen Statik,

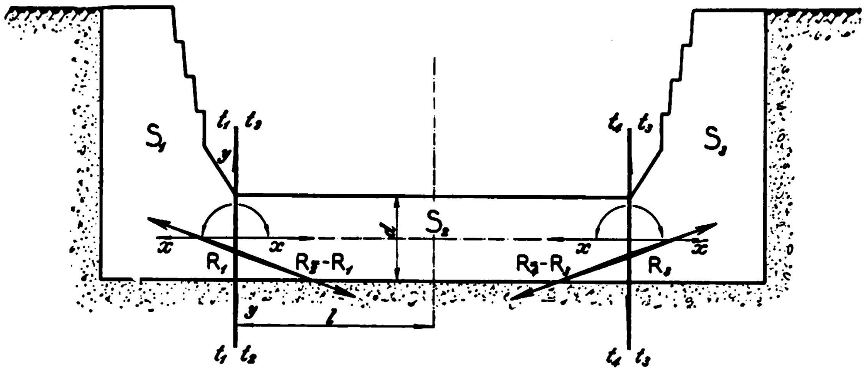

Fig. 11.

Dry dock Naples: Sketch cross section.

4. Teil: Der Bogen, Zürich, 1906 — p. 228). In the case of the beam it is necessary to make use of the theory of the elastically bedded beam. From considerations of symmetry it may be assumed that none but vertical displacements can occur at the central plane of the dock, so the elastic displacements or rotations of the end cross section due to unit longitudinal or transverse stress, or unit moment, are given by the following expressions, wherein the suffix x denotes a longitudinal, and y a transverse force or displacement, and x denotes a moment or rotation.

$$\left.\begin{aligned}
\beta_{xx} &= \frac{1}{EF} \\
\beta_{yx} &= \beta_{xy} = 0 \\
\beta_{zx} &= \beta_{xz} = (0) \\
\beta_{yy} &= \frac{z}{sC} \cdot \rho_3(\lambda) \\
\beta_{zy} &= \beta_{yz} = -\frac{2}{s^2C} \cdot \rho_2(\lambda) \\
\beta_{zz} &= \frac{4}{s^3C}\,\rho_1(\lambda)
\end{aligned}\right\}\ \text{where}\ \left\{\begin{aligned}
s &= \sqrt[4]{\frac{4Ei}{C}} \\
i &= \frac{d^3}{12} \\
\lambda &= \frac{l}{s} \\
\rho_1(\lambda) &= \frac{\mathrm{Ch}\,2\lambda - \cos 2\lambda}{\mathrm{Sh}\,2\lambda + \sin 2\lambda} \\
\rho_2(\lambda) &= \frac{\mathrm{Sh}\,2\lambda - \sin 2\lambda}{\mathrm{Sh}\,2\lambda + \sin 2\lambda} \\
\rho_3(\lambda) &= \frac{\mathrm{Ch}\,2\lambda + \cos 2\lambda}{\mathrm{Sh}\,2\lambda + \sin 2\lambda}
\end{aligned}\right.$$

The significance of d and l may be understood from Fig. 11; C is the ground constant (bedding number); E the modulus of elasticity of the material.

The elements of the ellipse are then given by the following expressions:

$$x_G = -\frac{\beta_{zy}}{\beta_{zz}}; \quad y_G = 0; \quad g = \beta_{zz}$$

$$i_1^2 = \frac{\beta_{yy}\,\beta_{zz} - \beta_{zy}^2}{\beta_{zz}^2}; \quad i_2^2 = \frac{1}{E\,d\,\beta_{zz}}$$

The condition that the dividing planes must undergo no mutual displacement can be expressed by constructing a resultant ellipse in which the elastic resistances of the two parts appear as if summed. By analogy with electrical theory, the two ellipses may then be regarded as connected in parallel.

If the elements of the two part-ellipses E_a and E_b are marked by the suffixes a and b, then if the major axes of the two ellipses are parallel to one another (as in the present case) the corresponding elements in the combined ellipse will be determined by the relationships

$$\frac{1}{g_a} + \frac{1}{g_b} = \frac{1}{g} \text{ und } \frac{1}{\lambda_a} + \frac{1}{\lambda_b} = \frac{1}{\lambda} \text{ wenn } \lambda_{ab} = i_1^2\,g_a, \text{ bezw.} = i_2^2\,g_b$$

and the position of the combined ellipse is fixed by the condition that the new major axis will divide the distance separating those of the part — ellipses in the same proportion as the corrsponding values of λ.

If the combined ellipse is established in this way no difficulty will arise in determining the magnitude of the forces taken up by the elastic bedding of the block and the magnitude transferred to the central beam across the section considered, in reference to any given condition of loading or to any given resultant of the external forces acting upon the side block.

The method of construction, as described above, allows settlement of the side walls and the dock floor to take place independently of one another. This being so, it is necessary in the present case to take account only of those external forces which will arise a f t e r the two longitudinal gaps have been filled — that is to say the weight of the masonry lining which is to be built after the dock is pumped out, the earth pressure of the back-fill along the sides, and (for the case where the dock space is pumped dry) the lateral water pressure not balanced by internal pressure; also the uplift acting under the floor.

In order to carry out these calculations, the effect due to the external load p_o acting on the floor (or on the elastic central beam) must be replaced by a virtual force P, also vertical, acting on the side block and passing through its centre of elasticity. The magnitude and direction of P are determined by the condition that this force must cause the same movement of the side block in relation to the central beam as is caused by the loading p_0 for which it is substituted; in other words

$$P \cdot i_{1b}^2 \cdot g_b = \frac{-p_o}{C}$$

where i_{1b} and g_b respectively denote the horizontal diameter and elastic weight of the ellipse appertaining to the side block, and C represents the ground constant. P is combined with the external forces which act directly on the side block to give the resultant R.

To divide R between the two components mentioned above, its anti-polar A is determined by reference to the combined ellipse. Then the lines of action of the partial forces appertaining to the side block and the central beam are the anti polars r_a and r_b of the point A by reference to the two partellipses.

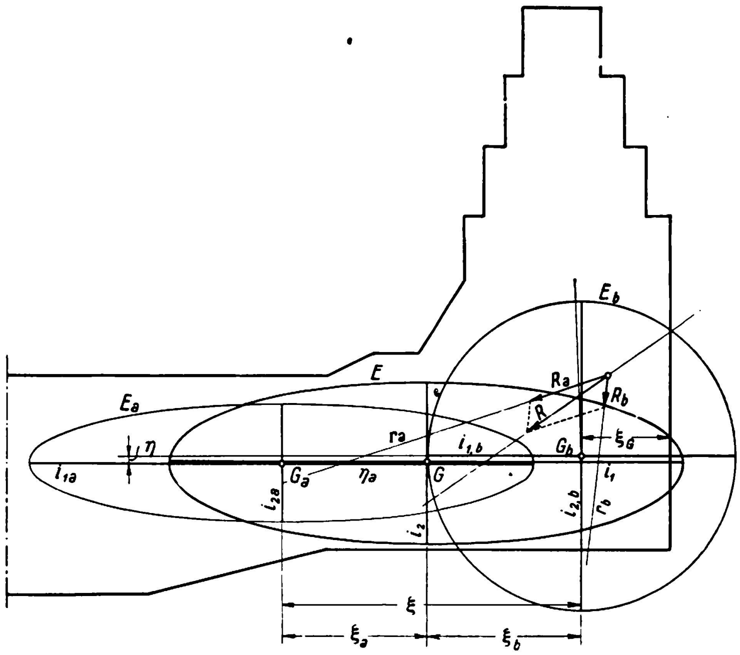

Fig. 12.

Dry dock Naples: Ellipses of elasticity.

The point of intersection of r_a and r_b falls on the line of action of R, which can thus be replaced at once by the two desired component forces, so solving the problem. (See Fig. 12.)

Determination of the ground constant C.

It is clear from the considerations developed above that the ground constant or bedding number C is of outstanding importance. As its value is known to be dependent on the size of the loaded area, and no experimental results are available for such large areas as here, the Società Italiana per Lavori Marittimi who were making the design decided on a large scale experiment. For this purpose use was made of the "Principe di Piemonte" dry dock in Venice, which is of similar dimensions to the new dock in Naples and rests, like the latter, on a sandy bottom.

The elastic movements of the dock, while being repeatedly filled and emptied, were measured at five points with the aid of a telescopic level magnifying 80 times, set up at a suitable distance so as not to participate in the movements of the dock (see Fig. 13). One of the movement diagrams obtained by this means (representative of the others which were all more or less similar) is shown in Fig. 14. The bedding numbers taken from the diagram vary between extreme values of 0.55 and 0.95 kg/cm^3.

On the basis of these experiments the value C = 0.75 kg/cm^3 = 750 t/m^3 was adopted for the statical examination of the dock at Naples.

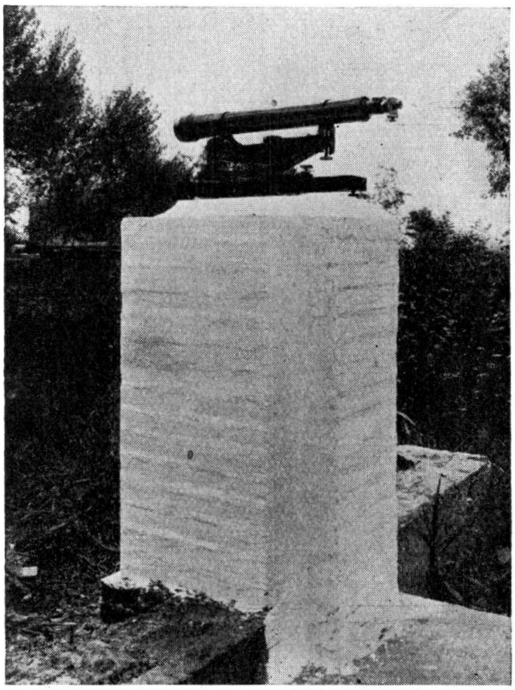

Fig. 13.

Observation station for determining the elastic movements of the dry dock "Principe di Piemonte" in Venice.

In view of the great importance which attaches to knowledge of this ground constant or bedding figure in projecting and designing hydraulic works of large size, such as dry docks, it would be a matter for congratulation if similar experiments were to be carried out elsewhere and their results published.

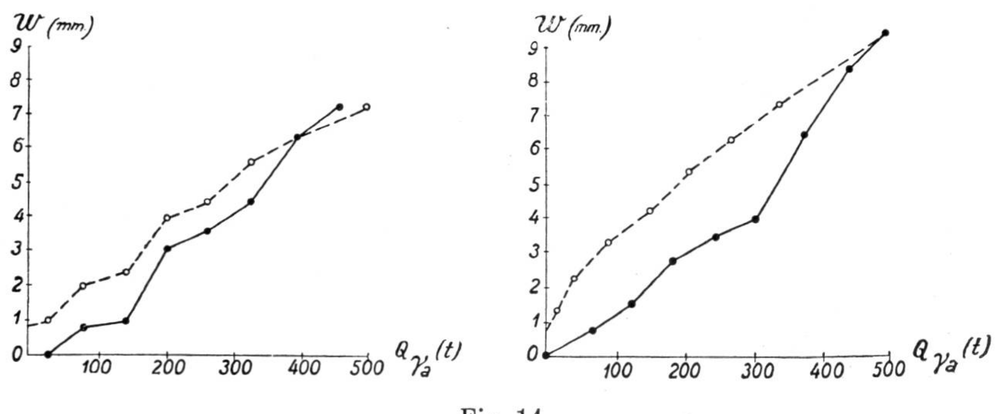

Fig. 14.

Movements of the dry dock "Principe di Piemonte" during filling and subsequent emptying.

Summary.

The Authors describe in their article the recently constructed dry docks of Genoa and Naples. The static behaviour is shown, which is fundamentally different in both cases. The dry dock of Genoa is founded on rock and the dry dock of Naples on sandy ground. To determine the "soil constant" test measurements were taken at the existing dry dock "Principe di Piemonte" in Venice.

Leere Seite
Blank page
Page vide